Chapter 2
Seismic Analysis of Concrete Hydraulic Structures

2-1. Introduction

a. General. This chapter provides structural guidance for the use of response spectra for the seismic design and evaluation of the Corps of Engineers hydraulic structures. These include locks, intake towers, earth retaining structures, arch dams, conventional and RCC gravity dams, powerhouses, and critical appurtenant structures. The specific requirements are provided for the structures built on rock, such as the arch and most gravity dams, as well as for those built on soil or pile foundations, as in the case of some lock structures. The response spectrum method of seismic design and evaluation provisions for building-type structures are summarized in paragraph 2-10.

b. Interdisciplinary collaboration. A complete development and use of response spectra for seismic design and evaluation of hydraulic structures require the close collaboration of a project team consisting of several disciplines.

(1) Project team. The specialists in the disciplines of seismology, geophysics, geology, and geotechnical engineering develop design earthquakes and the associated ground motions, with the results presented and finalized in close cooperation with structural engineers. The materials engineer and geotechnical specialists specify the material properties of the structure and of the soils and rock foundation. The structural engineer in turn has the special role of explaining the anticipated performance and the design rationale employed to resist the demands imposed on the structure by the earthquake ground motions.

(2) Ground motion studies. As discussed in Chapter 3, the seismic input in the form of site-specific response spectra is developed using a deterministic or a probabilistic approach. Both methods require the following three main items to be clearly addressed and understood so the project team members have a common understanding of the design earthquakes: seismic sources, i.e., faults or source areas that may generate earthquakes; maximum earthquake sizes that can occur on the identified sources and their frequency of occurrence; and attenuation relationships for estimation of ground motions in terms of magnitude, distance, and site conditions. The results of ground motion studies should be presented as required in ER 1110-2-1806. For a DSHA mean and 84^{th} percentile, response spectra for the MCE should be presented. For a PSHA, response spectra should be presented as equal hazard spectra at various levels of probability and damping, as described in ER 1110-2-1806 and Chapter 3. Acceleration time-histories based on natural or synthetic accelerograms may also be required. The assumptions and methodology used to perform a DSHA and PSHA should be explained, and the uncertainties associated with the selection of input parameters should be presented in the report.

2-2. General Concepts

Two essential problems must be considered in the seismic analysis and design of structures: definition of the expected earthquake input motion and the prediction of the response of the structure to this input. The solutions to these problems are particularly more involved for the structures founded on soil or pile foundations and for those built on rock sites with complicated topography as in the case of arch dams.

a. Input motion(s). A general description of the factors affecting the earthquake input motions to be used in the design and evaluation of structures is demonstrated in Figure 2-1. The base rock motion

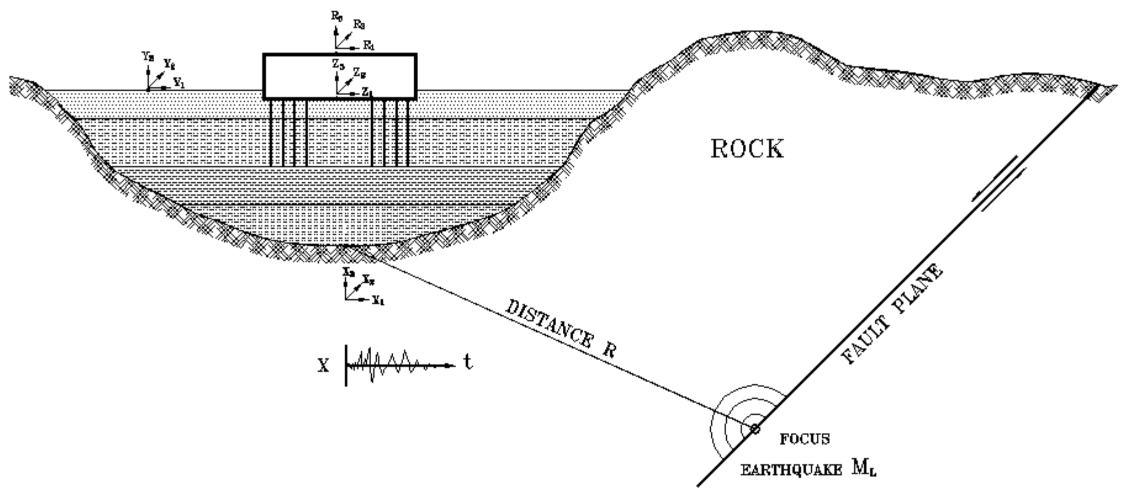


Figure 2-1. Factors affecting seismic input motion for a structure founded on soil-pile foundation

X_i(i = 1,2,3) is estimated from the study of regional geologic setting, historic seismicity of the area, and the geologic structure along the path from source to site. The characteristics of this motion, however, are affected by the local soil conditions as it travels to the free ground surface. Thus, the resulting free-field motion Y_i (i = 1,2,3), in the absence of the structure, differs from X_i in terms of the peak amplitude, the frequency content, and the spatial distribution of the motion characteristics. In addition, the dynamic interaction of the structure with the soil foundation produces a further change of the seismic motions, leading to Z_i (i = 1,2,3) at the soil-structure interface. Depending on the method of analysis adopted, one of these motions is selected as the earthquake input in the actual dynamic analysis of the structure. If X_i or Y_i is selected, the soil foundation is modeled as part of the structure, and a direct method of soil-structure interaction (SSI) analysis is performed. Alternatively, the structure and the soil region may be treated as two separate substructures. First the soil region is analyzed with the mass of the structure set to zero, to obtain ground motion Z_i at the soil-structure interface (kinematic interaction). The same model is also used to determine the dynamic stiffness of the soil region. Then Z_i is used as the input motions in the subsequent earthquake response analysis of the structure whose stiffness is now being combined with the dynamic stiffness of the soil region, and its mass being considered. To estimate these ground motions, however, many aspects of the problem such as the seismic environment, dynamic soil properties, site response, and the structural analysis must be considered. The solution thus requires close cooperation among the geologist, seismologist, and geotechnical and structural engineers to achieve satisfactory results.

b. Structural response. The second problem involves prediction of the response of the structure to the specified input motion. This requires development of a structural model, specification of material properties and damping, and calculation of the response, taking into account the dynamic interactions with the foundation, the water, and the backfill soils. Depending on complexity of the structure and intensity of the earthquake, a simple or more advanced modeling and analysis may be required. In either case the analysis should consist of the following steps, except that the level of effort may be different for simple and more refined analyses:

(1) Establishment of earthquake design criteria.

(2) Development of design earthquakes and associated ground motions.

(3) Establishment of analysis procedure.

(4) Development of structural models.

(5) Prediction of earthquake response of the structure.

(6) Interpretation and evaluation of results.

2-3. Design Criteria

The design and evaluation of hydraulic structures for earthquake loading must be based on appropriate criteria that reflect both the desired level of safety and the choice of the design and evaluation procedures (ER 1110-2-1806). The first requirement is to establish design earthquake ground motions to be used as the seismic input by giving due consideration to the consequences of failure and the designated operational function. Then the response of the structure to this seismic input must be calculated taking into account the significant interactions with the rock, soil, or pile foundation as well as with the impounded, or surrounding and contained water. The analysis should be formulated using a realistic idealization of the structure-water-foundation system, and the results are evaluated in view of the limitations, assumptions, and uncertainties associated with the seismic input and the method of analysis.

2-4. Design Earthquakes

a. Operating basis earthquake (OBE).

(1) Definition and performance. The OBE is an earthquake that can reasonably be expected to occur within the service life of the project, that is, with a 50 percent probability of exceedance during the service life. (This corresponds to a return period of 144 years for a project with a service life of 100 years.) The associated performance requirement is that the project function with little or no damage, and without interruption of function. The purpose of the OBE is to protect against economic losses from damage or loss of service. Therefore alternative choices of return period for the OBE may be based on economic considerations. In a site-specific study the OBE is determined by a PSHA (ER 1110-2-1806).

(2) Analysis. For the OBE, the linear elastic analysis is adequate for computing seismic response of the structure, and the simple stress checks in which the predicted elastic stresses are compared with the expected concrete strength should suffice for the performance evaluation. Structures located in regions of high seismicity should essentially respond elastically to the OBE event with no disruption to service, but limited localized damage is permissible and should be repairable. In such cases, a low to moderate level of damage can be expected, but the results of a linear time-history analysis with engineering judgment may still be used to provide a reasonable estimate of the expected damage.

b. Maximum design earthquake (MDE).

(1) Definition and performance. The MDE is the maximum level of ground motion for which a structure is designed or evaluated. The associated performance requirement is that the project performs without catastrophic failure, such as uncontrolled release of a reservoir, although severe damage or economic loss may be tolerated. The MDE can be characterized as a deterministic or probabilistic event (ER 1110-2-1806).

(a) For critical structures the MDE is set equal to the MCE. Critical structures are defined in ER 1110-2-1806 as structures of high downstream hazard whose failure during or immediately following an earthquake could result in loss of life. The MCE is defined as the greatest earthquake that can reasonably be expected to be generated by a specific source on the basis of seismological and geological evidence (ER 1110-2-1806).

(b) For other than critical structures the MDE is selected as a lesser earthquake than the MCE, which provides for an economical design meeting specified safety standards. This lesser earthquake is chosen based upon an appropriate probability of exceedance of ground motions during the design life of the structure (also characterized as a return period for ground motion exceedance).

(2) Nonlinear response. The damage during an MDE event could be substantial, but it should not be catastrophic in terms of loss of life, economics, and social and environmental impacts. It is evident that a realistic design criterion for evaluation of the response to damaging MDEs should include nonlinear analysis, which can predict the nature and the extent of damage. However, a complete and reliable nonlinear analysis that includes tensile cracking of concrete, yielding of reinforcements, opening of joints, and foundation failure is not currently practical. Only limited aspects of the nonlinear earthquake response behavior of the mass concrete structures such as contraction joint opening in arch dams, tensile cracking in concrete gravity dams, and sliding of concrete monoliths have been investigated previously. There is a considerable lack of knowledge with respect to nonlinear response behavior of the hydraulic structures. Any consideration of performing nonlinear analysis for hydraulic structures should be done in consultation with CECW-ED.

(3) Performance evaluation. The earthquake performance evaluation of the response of hydraulic structures to a damaging MDE is presently based on the results of linear elastic analysis. In many cases, a linear elastic analysis can provide a reasonable estimate of the level of expected damage when the cracking, yielding, or other forms of nonlinearity are considered to be slight to moderate.

(a) URC. For URC hydraulic structures subjected to a severe MDE, the evaluation of damage using the linear time-history analysis may still continue. The evaluation, however, must be based on a rational interpretation of the results by giving due consideration to several factors including number and duration of stress excursions beyond the allowable limits, the ratio of computed to allowable values, simultaneous stress distributions at critical time-steps, size and location of overstressed area, and engineering judgment.

(b) RC. Such evaluation for the RC hydraulic structures should include approximate postelastic analysis of the system considering ductility and energy dissipation beyond yield. First the section forces for critical members are computed using the linear elastic analysis procedure described in this manual. These forces are defined as the force *demands* imposed on the structure by the earthquake. Next the yield or plastic capacities at the same locations are computed and defined as the force *capacities*. Finally, the ratio of force demands to force capacities is computed to establish the *demand-capacity ratios* for all the selected locations. The resulting demand-capacity ratios provide an indication of the ductility that may be required for the structural members to withstand the MDE level of ground motion. If the computed demand-capacity ratios for a particular structure exceed the limits set forth in the respective design documents for that structure, approximate postelastic analyses should be performed to ensure that the inelastic demands of the MDE excitation on the structure can be resisted by the supplied capacity. This evaluation consists of several equivalent linear analyses with revised stiffness or resistance characteristics of all structural members that have reached their yielding capacities. The stiffness modification and analysis of the modified structure are repeated until no further yielding will occur or the structure reaches a limit state with excessive distortions, mechanism, or instability.

2-5. Earthquake Ground Motions

Earthquake ground motions for analysis of hydraulic structures are usually characterized by peak ground acceleration, response spectra, and acceleration time-histories. The peak ground acceleration (usually as a fraction of the peak) is the earthquake ground motion parameter usually used in the seismic coefficient method of analysis. The earthquake ground motions for dynamic analysis, as a minimum, should be specified in terms of response spectra (Figure 2-2). A time-history earthquake response analysis, if required, should be performed using the acceleration time-histories. The standard response spectra are described in the following paragraphs, and procedures for estimating site-specific response spectra are discussed in Chapter 3.

a. Elastic design response spectra. Elastic design response spectra of ground motions can be defined by using standard or site-specific procedures. As illustrated in Figure 2-2, elastic design response spectra represent maximum responses of a series of single-degree-of-freedom (SDOF) systems to a given ground motion excitation (Ebeling 1992; Chopra 1981; Clough and Penzien 1993; Newmark and Rosenblueth 1971). The maximum displacements, maximum pseudo-velocities, and maximum pseudo-accelerations presented on a logarithmic tripartite graph provide advance insight into the dynamic behavior of a structure. For example, Figure 2-2 shows that at low periods of vibration (<0.05 sec), the spectral or pseudo-accelerations approach the PGA, an indication that the rigid or very short period structures undergo the same accelerations as the ground. This figure also shows that structures with periods in the range of about 0.06 to 0.5 sec are subjected to amplified accelerations and thus higher earthquake forces, whereas the earthquake forces for flexible structures with periods in the range of about 1 to 20 sec are reduced substantially but their maximum displacements exceed that of the ground. In the extreme, when the period exceeds 20 sec, the structure experiences the same maximum displacement as the ground. The response spectrum amplifications depend on the values of damping and are significantly influenced by the earthquake magnitude, source-to-site distance, and the site conditions (Chapter 3).

(1) Standard or normalized response spectra. The standard response spectra described in this section are to be used in accord with ER 1110-2-1806 and follow-up guidance. The development starts with the spectral acceleration ordinates obtained from the National Earthquake Hazards Reduction Program (NEHRP) hazard maps. The site coefficients to be used with the hazard maps to develop standard response spectra for various soil profiles, as well as the methodology to construct response spectra at return periods other than those given in NEHRP, are provided in the guidance in ER 1110-2-1806.

(2) Site-specific response spectra.

(a) Site-specific procedures to produce design response spectra are to be used in accord with ER 1110-2-1806. Site-specific response spectra correspond to those expected on the basis of the seismological and geological calculations for the site. The procedures described in Chapter 3 use either the deterministic or probabilistic method to develop site-specific spectra.

(b) While the deterministic method provides a single estimate of the peak ground acceleration and response spectral amplitudes, the probabilistic method estimates these parameters as a function of probability of exceedance or return period. To select the return period to use for the OBE and MDE, see the definitions of these design earthquakes in paragraphs 2-4a and 2-4b, respectively. The resulting response spectra for the selected return period should then be used as input for quantifying the seismic loads required for the design and analysis of structures.

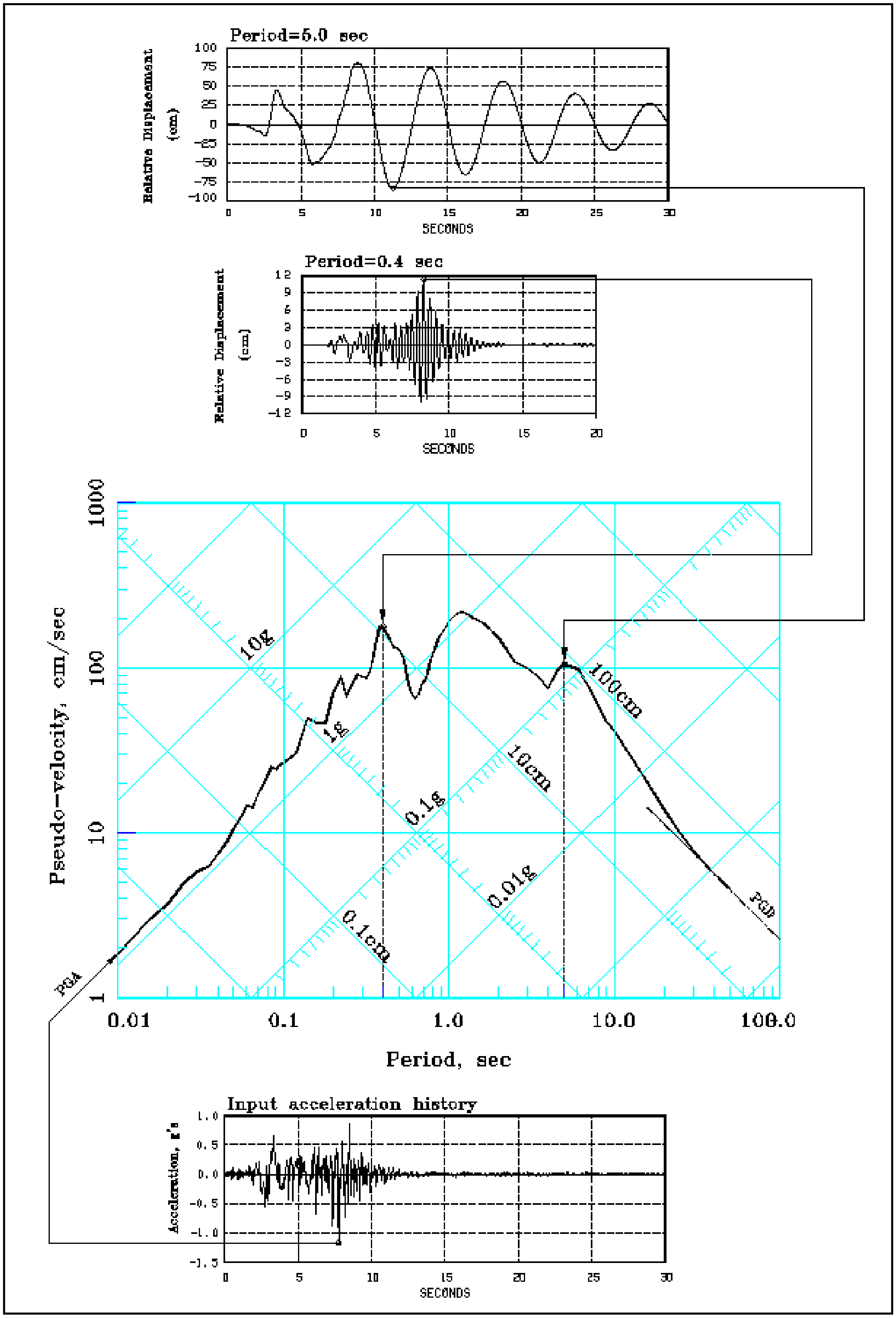


Figure 2-2. Construction of tripartite elastic design response spectrum

b. Acceleration time-histories. Various procedures for developing representative acceleration time-histories at a site are described in Chapter 3. Whenever possible, the acceleration time-histories should be selected to be similar to the design earthquake in the following aspects: tectonic environment, earthquake magnitude, fault rupture mechanism (fault type), site conditions, design response spectra, and duration of strong shaking. Since it is not always possible to find records that satisfy all of these criteria, it is often necessary to modify existing records or develop synthetic records that meet most of these requirements.

2-6. Establishment of Analysis Procedures

Seismic analysis of hydraulic structures should conform to the overall objectives of new designs and satisfy the specific requirements of safety evaluation of existing structures. The choice of analysis procedures may influence the scope and nature of the seismic input characterization, design procedures, specification of material properties, and evaluation and interpretation of the results. Simple procedures require fewer and easily available parameters, while refined analyses usually need more comprehensive definition of the seismic input, structural idealization, and material properties. The analysis should begin with the simplest procedures possible and then, if necessary, progress to more refined and advanced types. Simplified procedures are usually adequate for the feasibility and preliminary studies, whereas refined procedures are more appropriate for the final design and safety evaluation of structures. The simplified analysis also serves to assess the need for a more elaborate analysis and provide a baseline for comparison with the results obtained from the more elaborate analyses.

2-7. Structural Idealization

Structural models should be developed by giving careful consideration to the geometry, stiffness, and mass distributions, all of which affect the dynamic characteristics of the structure. The engineering judgment and knowledge of the dynamics of structures are required to develop a satisfactory model that is both simple and representative of the most important dynamic behavior of the structure. Depending on its level of complexity, a hydraulic structure may be represented by a simplified one-dimensional model, a planar or 2-D model whose deformations are restricted in a plane, or by a more elaborate 3-D model to account for its 3-D behavior.

a. Simplified models. Structures with regular geometry and mass distribution along one axis may be idealized by simplified models using the beam theory. The simplified model should approximately represent the significant features of the dynamic response of the structure including the fundamental period and mode shape, as well as the effects of structure-foundation and structure-water interaction. Two such simplified models have been developed for the free-standing intake towers and the nonoverflow gravity dam sections. In both cases, the simplified models were formulated based on the results of finite element analyses that rigorously accounted for the structure-water-foundation interaction effects, as well as for the reservoir bottom energy absorption for the gravity dams.

(1) Simplified model for intake towers. The preliminary design and safety evaluation of the free-standing and regular intake towers may be conducted using the simplified model shown in Figure 2-3. A step-by-step analysis procedure for this cantilever beam model is provided in Goyal and Chopra (1989) and Appendix H. Some important features and assumptions of this approximate model are as follows:

(a) It is applicable to towers with regular geometry in plan and elevation.

(b) Only flexural deformations are considered.

(c) Seismic response is calculated for the first two modes of vibration.

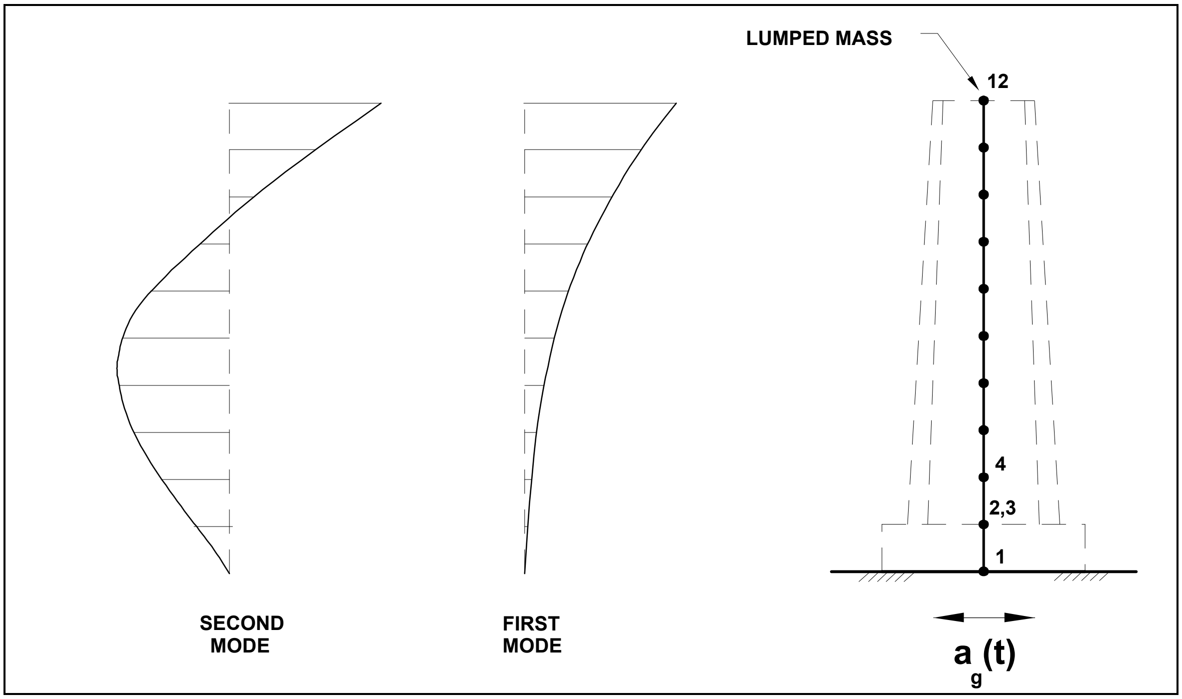


Figure 2-3. Simplified model of intake tower ($a_g(t)$ = ground motion acceleration)

(d) Foundation-structure interaction effects are considered only for the first mode of vibration.

(e) Interaction between the tower and the inside and outside water is represented by the added mass assumption.

(f) The effects of vertical component of ground motion are ignored.

Note that slender towers with cross-section dimensions 10 times less than the height of the structure can usually be adequately represented solely by the flexural deformations of the tower. However, the effects of shear deformations on vibration frequencies and section forces, especially for higher modes, are significant when the cross-section dimensions exceed 1/10 of the tower height and should be included in the analysis. The effects of shear deformation can be incorporated in the analysis if a computer program with beam elements including shear deformation is used. The earthquake response for this simplified model should be calculated for the combined effects of the two horizontal components of the ground motions. The maximum shear forces, moments, and stresses for each lateral direction are computed separately using the specified response spectrum and the calculated vibration properties associated with that direction. The total response values of the tower are then obtained by combining the responses caused by each of the two components of the earthquake ground motion, as discussed in paragraph 2.8a(2)(f).

(2) Simplified model for gravity dams. The preliminary design and safety evaluation of gravity dams may start with a simplified model developed by Fenves and Chopra (1986), as shown in Figure 2-4. In this procedure, deformations of the dam monolith are restricted to the fundamental mode of vibration of the dam on rigid foundation rock. Standard values are provided for the fundamental vibration period and mode shape of typical nonoverflow gravity sections. But they are not available for the nonstandard or spillway sections whose geometries substantially differ from that of a typical nonoverflow section. In such cases, the fundamental vibration period and mode shape for the nonstandard section should be

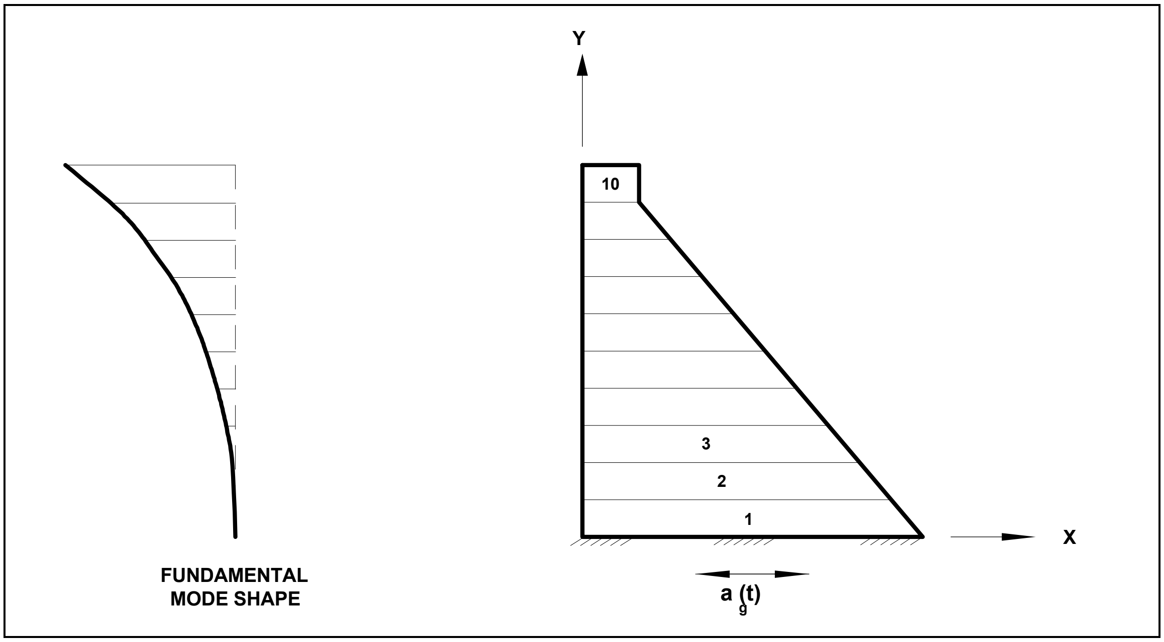


Figure 2-4. Simplified model of gravity dam monolith (Fenves and Chopra (1986), courtesy of Earthquake Engineering Research Center, University of California at Berkeley)

estimated using other procedures before this simplified method could be applied. The most important features of the simplified model are summarized as follows.

(a) Only fundamental mode of vibration is considered; contributions due to higher modes are accounted for by static correction.

(b) The upstream face of the dam is assumed to be vertical or nearly vertical.

(c) The effects of vertical ground motion are ignored.

(d) The interactions with the foundation rock and water are accounted for by adjustment of the vibration period and damping of the fundamental mode. The inertial effects of the added mass of water are considered in terms of additional lateral hydrodynamic forces.

b. Two-dimensional models. 2-D idealization is used to model planar or very long structures. Most Corps of Engineers hydraulic structures are of the latter type such as the retaining walls, gravity dams, outlet tunnels, and lock structures. These structures are usually made of independent segments separated by construction joints, and the loads perpendicular to the long axis are assumed not to vary along each segment. Under these conditions, the structure may be modeled as a 2-D slice using either the plane stress or plane strain elements, as shown in Figure 2-5. The choice of plane stress or plane strain elements depends on whether the stress or strain in the out-of-plane direction can be neglected. In either case, plane strain models should be used to idealize the foundation supporting the structure. 2-D models should be analyzed for two components of the earthquake ground motion applied in the vertical and one horizontal direction. However, the way the seismic input is applied to the structure depends on the type of foundation model being used. Three commonly used foundation models and their associated seismic input for the analysis of typical hydraulic structures are discussed in the following paragraphs.

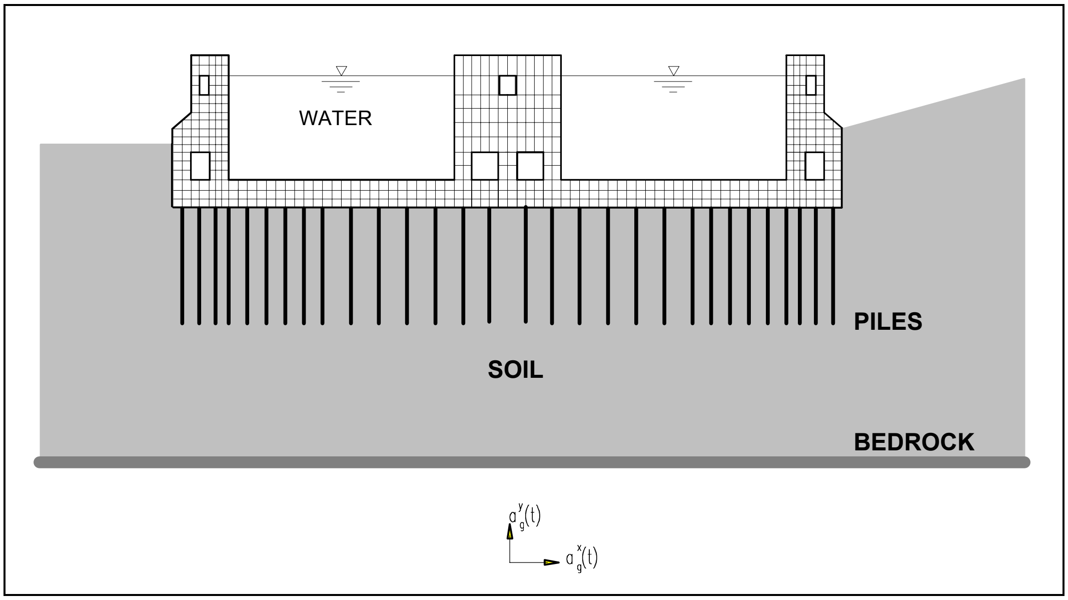


Figure 2-5. 2-D model of W-frame lock

(1) Rigid rock-base excitation. The standard approach to accounting for the effects of the foundation interaction is to analyze the combined structure-foundation system by including an appropriate region of the rock in the finite element idealization, as shown in Figure 2-6a. In this approach, the earthquake motion is represented as a rigid body translation a_R of the basement rock, and either the response spectra or acceleration time-histories are used as input to the model. The characteristics of the specified earthquake ground motion should be similar to the motions recorded on the rock sites. The location of the rigid boundary at the base of the model should be selected consistent with the size and type of the structure being analyzed. The mass of foundation rock should be ignored so that the free-field motions recorded at ground surface are directly applied to the structure without changes, and the spurious reflection effects caused by the rigid boundary assumption are eliminated.

(2) Free-field earthquake excitation. For rock and firm soil sites where similar foundation materials extend to large depths, the foundation region may be idealized as a homogeneous, isotropic, viscoelastic

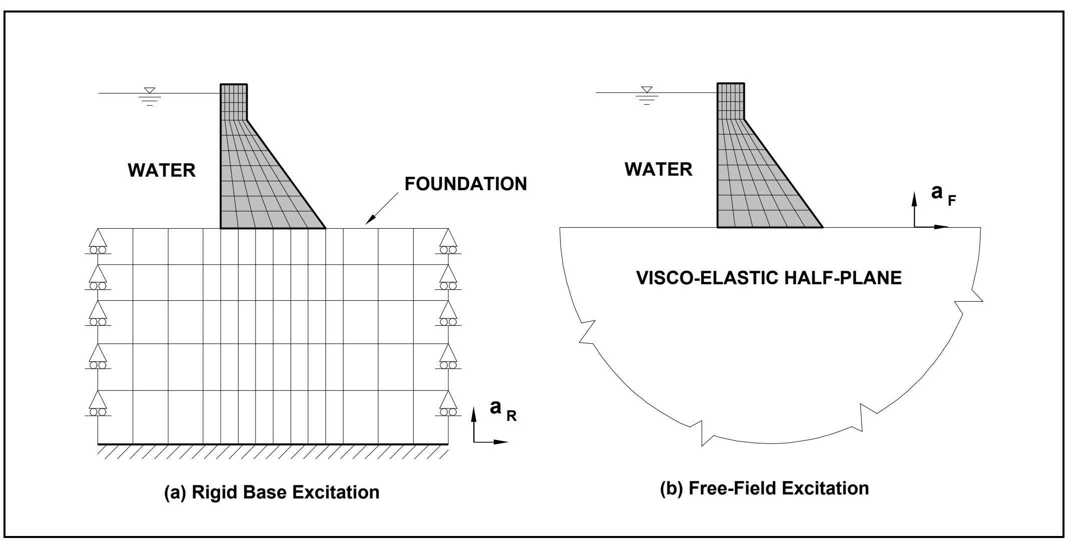


Figure 2-6. Earthquake excitation for rock or firm soil sites

half-plane (Dasgupta and Chopra 1979), as shown in Figure 2-6b. In this case, the structure is supported on the horizontal surface of the foundation, and the earthquake response is formulated with respect to the free-field definition of the ground motion a_F rather than the basement rock input. The interaction effects of the foundation are represented by a frequency-dependent dynamic stiffness matrix defined with respect to the degrees of freedom on the structure-foundation interface. The seismic input for this idealization is in the form of acceleration time-histories of the free-field motion; the response spectrum method of analysis is not applicable. This method is currently used in the analysis of gravity dams and free-standing intake towers when the foundation material can be assumed homogeneous.

(3) Soil-pile-structure earthquake excitation. Unlike the gravity dams and intake towers, lock structures may be supported on pile groups embedded in nonhomogeneous soil media. In such cases, the soil-pile-structure interaction significantly affects the earthquake response of the structure and piles and should be considered in the analysis. Figure 2-7 schematically presents two methods for the earthquake analysis of structures founded on the soil-pile foundations (Wass and Hartmann 1984). In the direct method illustrated in Figure 2-7a, the piles and the soil up to the transmitting boundaries are modeled as part of the structure. The nonlinear soil behavior may be represented by the equivalent linear method (Seed and Idriss 1969). The seismic input in the form of acceleration time-histories is applied at the rock basement (rock-soil interface), and the earthquake response of the structure and the pile forces are determined. Alternatively, the analysis may be performed in two steps consisting of the kinematic and inertial parts, with the total motion *a* divided into a_k and a_i caused by kinematic and inertial interactions, respectively, as shown in Figures 2-7b and 2-7c. First the kinematic interaction is evaluated using the same model employed in the direct method, except that the mass of the structure is set to zero (M = 0). This analysis provides the ground motions a_k at the structure-soil interface, the required seismic input for the subsequent dynamic analysis for the inertial-interaction effects. The dynamic stiffness matrix of the soil-pile foundation needed for the inertial interaction analysis is also determined from the analysis of the same model employed in the kinematic interaction analysis. However, the resulting dynamic stiffness (or impedance function) for the soil-pile region is a complex valued matrix that requires solution in the frequency domain. The 2-D direct method and kinematic interaction analysis described above have been used for the analysis of pile foundation with backfill soils using the FLUSH program (Olmsted Locks and Dam, Design Memorandum No. 7, U.S. Army Engineer District, Louisville 1992). These analyses also provide response spectrum seismic input at the pile tips required for performing 3-D rigid-cap pile-group dynamic analysis using the Computer-Aided Structural Engineering (CASE) computer program, X0085 (CPGD), Dynamic Analysis of Pile Groups.

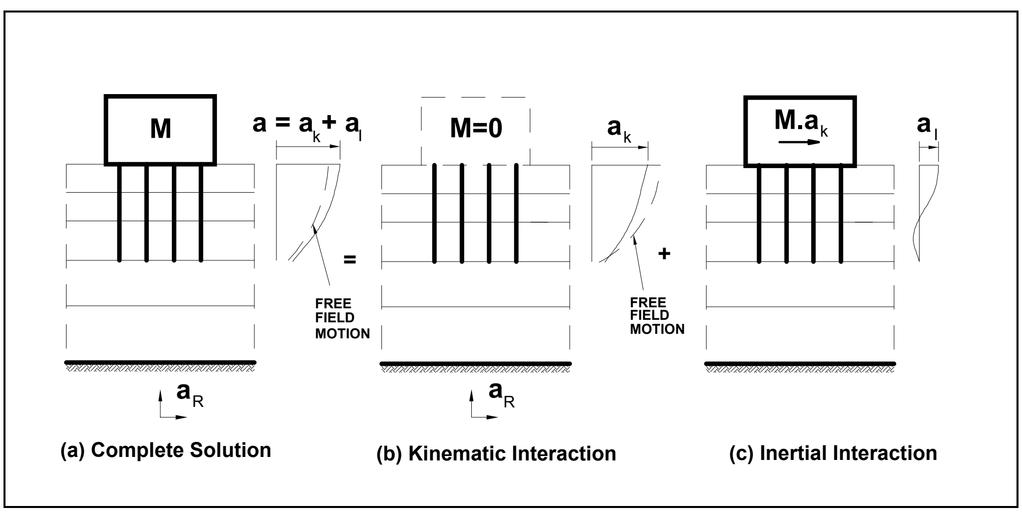


Figure 2-7. Schematic of earthquake response analysis for soil-pile-structure interaction

c. *Three-dimensional (3-D) models.* 3-D finite-element models are used to analyze hydraulic structures with complex geometry or nonuniform loading. Such structures include the arch dams, inclined intake towers supported on the abutment foundations, irregular free-standing intake towers with significant torsional behavior, gravity dams built in narrow canyons, and certain lock monoliths with complicated components and loading conditions. Arch dams must be treated as 3-D systems consisting of the concrete arch, foundation rock, and the impounded water (Figure 2-8). The inclined intake towers should be treated as 3-D structures to account for not only their complicated geometry and torsional behavior, but also for ground motions that must be applied at the tower base and along the entire tower-abutment interface. The irregular free-standing towers exhibiting dominant torsional modes of vibration should also be analyzed using 3-D models. Gravity dams built in narrow canyons are another example requiring 3-D treatment, because the customary assumption that dam monoliths behave independently is no longer valid—the movements of each monolith under these conditions are restrained by the adjacent ones, causing torsional moments or twists that would affect the manner in which the loads are distributed. The pile-founded lock structures with complicated geometry and structural components usually feature complicated soil-pile-structure interaction, which may require 3-D treatment. The specification of earthquake input for analysis of 3-D structures depends on the level of sophistication and capabilities used in modeling the dynamic behavior of the structure. The basic procedures are based on the general concepts described by Clough et al. (1985). These are summarized in the following paragraphs.

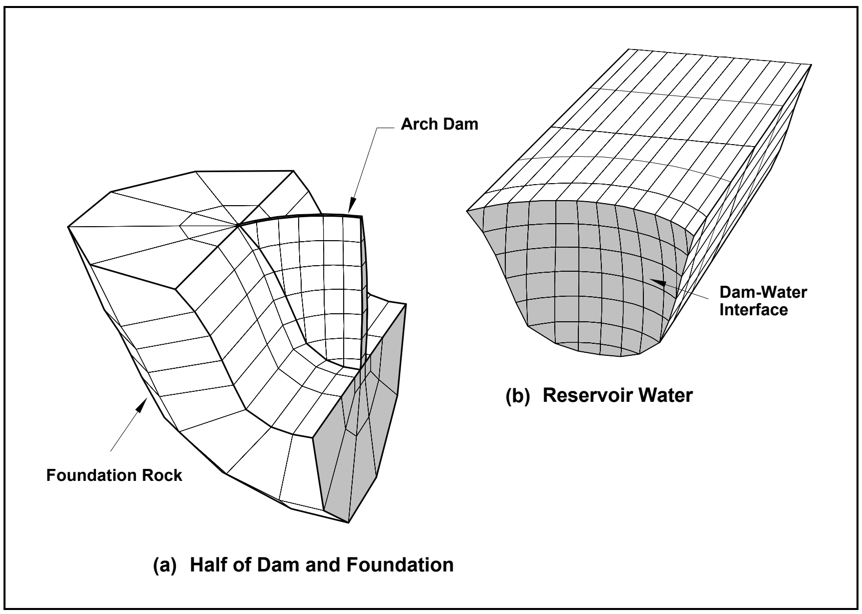


Figure 2-8. 3-D model of arch dam-water-foundation system

(1) Standard base input model. It is assumed that 3-D structures built on rock sites are supported by a large volume of deformable rock, which in turn is supported by a rigid boundary. The seismic input in the form of response spectra or acceleration time-histories is defined as the motion of this rigid base, but it should be noted that the motions applied to the rigid base differ from the free-field motions recorded at ground surface.

(2) Massless foundation rock model. An improved version of the model described in (1) above is obtained by neglecting the mass of the deformable foundation region. In this case no wave propagation takes place through the foundation rock; thus the prescribed motions at the rigid base are directly transmitted to the structure interface. With this assumption it is reasonable to use the earthquake motions

recorded at the ground surface as the rigid base input as for the 2-D analysis in Figure 2-6a. This procedure is commonly used in the practical analysis of 3-D structures built on rock sites. GDAP (Ghanaat 1993) and ADAP-88 (Fenves, Mojtahedi, and Reimer 1989) and other arch dam analysis programs commonly use this type of foundation model.

(3) Deconvolution base rock input model. In this approach the recorded free-field surface motions are deconvolved to determine the motions at the rigid base boundary. The deconvolution analysis is performed on a horizontally uniform layer of deformable rock or soil deposits using the one-dimensional wave propagation theory. For the soil sites, however, the strain-dependent nature of the nonlinear soil should be considered. The resulting rigid base motion is then applied at the base of the 3-D foundation-structure system, in which the foundation model is assumed to have its normal mass as well as stiffness properties. This procedure permits the wave propagation in the foundation rock, but requires an extensive model for the foundation rock, which computationally is inefficient.

(4) Free-field input model. A more reasonable approach for defining the seismic input would be to apply the deconvolved rigid base motion to a foundation model without the structure in place and to calculate the free-field motions at the interface positions, where the structure will be located. These interface free-field motions would be used as input to the combined structure-foundation model, which employs a relatively smaller volume of the rock region. It should be noted that the resulting seismic input at the interface varies spatially due to the scattering effects of canyon walls (in the case of arch dams) in addition to the traveling wave effects that also take place in the relatively long structures, even when the contact surface is flat. In either case, the computer program used should have capabilities to permit multiple support excitation. The application of this procedure has not yet evolved to practical problems.

(5) Soil-pile-structure interaction model. The seismic input for 3-D structures supported on pile foundations may be evaluated using a 3-D extension of the procedure discussed in b(3) above. However, the soil-pile-structure interaction analysis should also consider the inclined propagating body and surface waves if the structure is relatively long and is located close to a potential seismic source, or if it is supported on a sediment-filled basin. In particular, long-period structures with natural periods in the predominant range of surface waves should be examined for the seismic input that accounts for the effects of surface waves. One limiting factor in such analyses is the maximum number of piles that can be considered in the analysis of structures on a flexible base. For example a pile-founded lock structure may include a monolith having more than 800 piles. 3-D soil-structure interaction analysis programs such as SASSI (Lysmer et al. 1981) with pile groups analysis capability may not be able to handle such a large problem without some program modifications or structural modeling assumptions that could lead to a reduced number of piles for the idealized monolith.

2-8. Dynamic Analysis Procedures

The idealized model of structures and the prescribed earthquake ground motions are used to estimate the dynamic response of structures to earthquakes. The dynamic analysis is performed using the response spectrum or time-history method. The response spectrum method is usually a required first step in a dynamic analysis for the design and evaluation of hydraulic structures. In many cases it suffices for the structures located in low seismic hazard regions. It is also the preferred design tool, because the maximum response values for the design can be obtained directly from the earthquake response spectrum. However, the response spectrum procedure is an approximate method for calculating only the maximum response values and is restricted to the linear elastic analysis. The time-history method, on the other hand, is applicable to both linear elastic and nonlinear response analyses and is used when the time-dependent response characteristics or the nonlinear behavior is important, as explained later.

a. Modal analysis procedure. The modal superposition method is used to compute the earthquake response of structures within their linear elastic range of behavior. This procedure is especially applicable to the majority of Corps of Engineers hydraulic structures that are designed to remain essentially elastic when subjected to the medium intensity ground motions, such as the OBE. The modal analysis is also used for the MDE excitation, except that the computed linear elastic response is permitted to exceed the concrete cracking and yield stress levels for a limited amount in order to account for energy absorption of the structure. As illustrated in Figure 2-9, the primary feature of the modal analysis is that the total response of a structure is obtained by combining the response of its individual modes of vibration calculated separately. Furthermore, only the response in the first few modes need be calculated, because the response of structures to earthquakes is essentially due to the lower modes. The response of each individual mode is computed from the analysis of an SDOF system, according to the procedures described in the following paragraphs.

(1) Simplified Response Spectrum Analysis. The simplified response spectrum analysis (SRSA) is used for dynamic analysis of structures for which a simplified model of the types described in paragraph 2-7a can be developed. Whenever possible, this approximate analysis should be attempted to provide a preliminary estimate of the seismic response, as well as a basis for comparison with the results of a more refined analysis. The SRSA is normally employed for the analysis of structures whose dynamic behavior can be represented by an equivalent SDOF system. The maximum response of an idealized structure by the SRSA procedure is estimated as follows:

(a) Design response spectrum. For a preliminary analysis the standard response spectra described in paragraph 2-5a(1) should be used when a site-specific response spectrum does not exist.

(b) Natural frequencies and vibration mode shapes. Use the standard simplified procedures (Fenves and Chopra 1986, Goyal and Chopra 1989) to calculate the fundamental natural period and mode shape for the nonoverflow gravity dam sections and the regular intake towers. For other structures idealized by an equivalent SDOF system, the fundamental frequency and mode shape may be computed using the iterative methods described by Clough and Penzien (1993).

(c) Damping. Energy dissipation in the form of a damping ratio is included as part of the response spectrum curves. For the linear elastic or nearly elastic response during an OBE event, the damping value should be limited to 5 percent. For the MDE excitation, a damping constant of 7 or 10 percent may be used depending on the level of strains and the amount of inelastic response developed in the structure.

(d) Maximum modal displacement. The spectral acceleration, S_{an} $(T_n\ \xi_n)$ corresponding to the nth mode (here nth mode is assumed to be the fundamental mode) period of vibration, T_n, and the specified damping ratio, ξ_n, is directly obtained from the prescribed response spectrum. The maximum modal displacement in terms of S_{an} $(T_{n,}\ \xi_n)$ is given by:

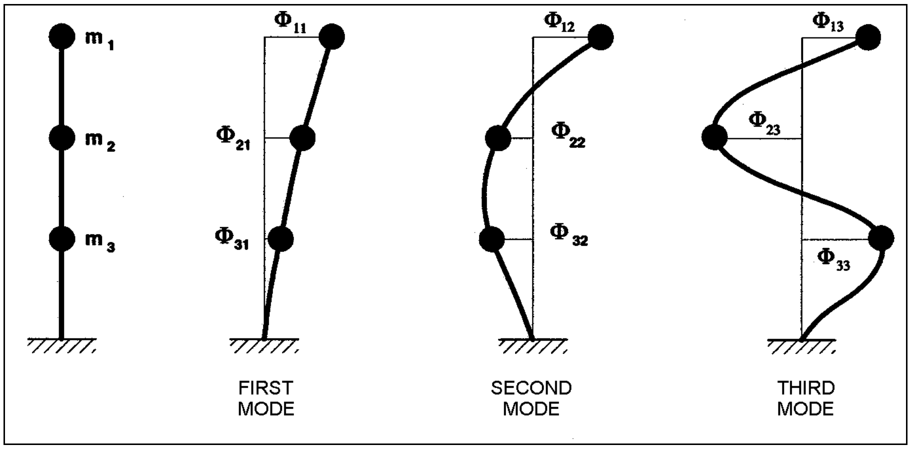
$$Y_n = \frac{L_n}{M_n \omega_n^2} S_{an}\left(T_n, \xi_n\right) \tag{2-1}$$

where

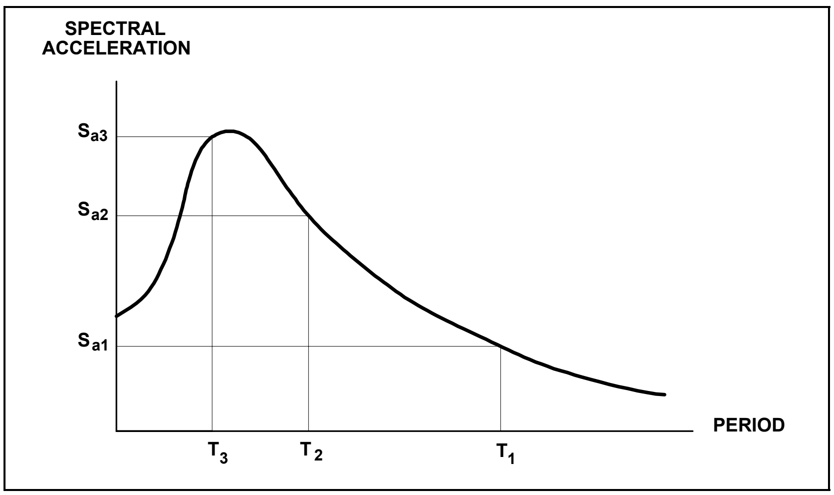
$L_n = \sum_{j=1}^{K} m_j \phi_{jn}$ is the modal earthquake-excitation factor

$M_n = \sum_{j=1}^{K} m_j \phi_{jn}^2$ is the modal mass

RESPONSE SPECTRUM ANALYSIS



(1) Compute mode shapes [ϕ_{j1}, ϕ_{j2}, ϕ_{j3}] and natural periods [T_1, T_2, T_3]



(2) Obtain spectral accelerations [S_{a1}, S_{a2}, S_{a3}] for all modes

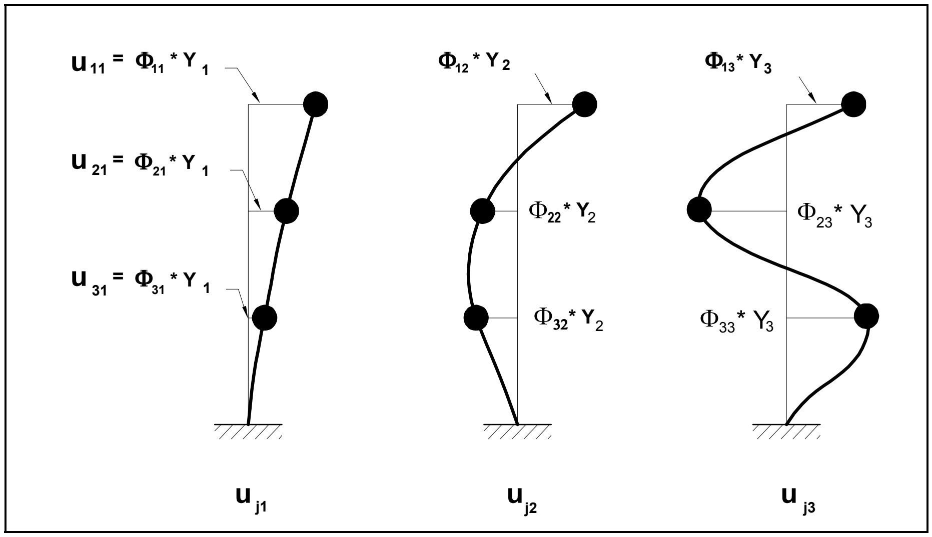
$$L_n = \sum_{j=1}^{3} \phi_{jn} m_j \quad ; \qquad M_n = \sum_{j=1}^{3} \phi_{jn}^2 m_j$$

$$PF_n = L_n / M_n$$

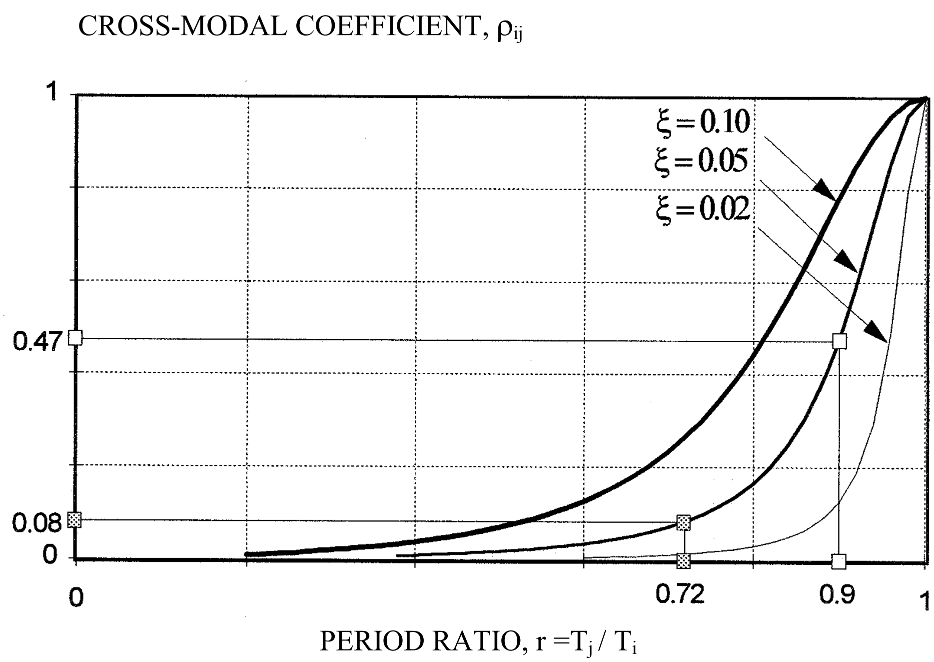
$$Y_n = \frac{PF_n}{\omega_n^2} \cdot S_{an} \qquad where \quad n = 1,2,3$$

(3) Compute modal participation factor PF_n
and maximum modal response Y_n

Figure 2-9. Illustration of response spectrum mode-superposition analysis (Continued)



(4) Compute maximum response of structure in each mode from $u_n = \Phi_{n*} Y_n$



(5) Obtain cross-modal coefficients ρ_{ij}

$$u_k = \left[\sum_{i=1}^{3} \sum_{j=1}^{3} u_{ki} \rho_{ij} u_{kj} \right]^{1/2}$$

(6) Compute total response u_k due to earthquake in direction *k*

Figure 2-9. (Concluded)

$\omega_n = 2\pi/T_n$ is the circular frequency

K = number of degrees of freedom in structural model

The ratio L_n/M_n is the modal participation factor PF_n indicating the degree to which mode n is excited by the ground motion.

(e) Maximum displacement. With the maximum modal displacement being computed in step (d), the maximum displacement of the structure is computed as

$$u_{jn} = \frac{L_n}{M_n \omega^2} \phi_{jn} S_{an}\left(T_n, \xi_n\right) \tag{2-2}$$

where ϕ_{jn} is the assumed or calculated mode shape of the structure.

(f) Maximum shear and moment. The shear forces and moments at sections along the height of the structure are obtained by static analysis from the equivalent lateral forces as follows:

$$f_{jn} = \frac{L_n}{M_n} m_j \phi_{jn} S_{an}\left(T_n, \xi_n\right) \tag{2-3}$$

$$V_n = \sum f_{jn} \tag{2-4}$$

$$M_n = \sum h_j f_{jn} \tag{2-5}$$

where

f_{jn} = maximum value of equivalent lateral force at the j^{th} section

h_j = associated moment arm

(2) Response spectrum modal superposition method. The estimation of maximum response of a hydraulic structure to earthquake excitation usually involves many modes of vibration, which may contribute significantly to the response. The contributions of various modes to the total displacements, forces, and stresses depend on a number of factors including the response spectrum ordinates, natural periods of vibration, and mode shapes, which in turn depend on the mass and stiffness properties of the structure. The seismic responses of such structures are further complicated by the dynamic interaction with the foundation supporting the structure and the impounded water. In general, the simplified SDOF procedures described above may not be applicable in most cases or may provide only a very crude estimate of the response. In these situations, the structure is analyzed using the response-spectrum modal-superposition method illustrated in Figure 2-9. The response-spectrum modal-superposition analysis is usually carried by standard or specialized programs following the same analysis steps described in a(1), but additional factors including the number of modes, combination of modal responses, and the effects of multiple components of earthquake input should also be considered.

(a) Number of modes. There are no guidelines for determining in advance how many modes should be included in a response spectrum analysis, because it depends on the dynamic characteristics of the structure and the response spectrum ordinates. However, the analysis should include a sufficient number of modes until the calculated response quantities are at least within 10 percent of the “exact” values. Since the “exact” response values are not known, a trial and error procedure may be adapted, in which analyses are repeated with addition of modes until it is seen that the addition of modes does not significantly affect the results. Alternatively, it may be demonstrated that the participating effective modal masses are at least within 90 percent of the total mass of the structures.

(b) Combination of modal responses. The response spectrum analysis procedure described above provides only the maximum response in each mode of vibration. The response quantities of interest, such as the peak displacements, element stresses, element forces, and moments, evaluated for each significant mode of vibration should be combined to obtain the total response of the structure. Since modal responses do not occur at the same time during the earthquake excitation, they should be combined using the complete quadratic combination (CQC) or the square root of the sum of the squares (SRSS) method described below.

(c) CQC method. The CQC modal combination method (Wilson, Der Kiureghian, and Bayo 1981) is based on random vibration theory and can be used in the response spectrum analysis if the duration of the strong motion portion of the earthquake shaking is several times longer than the fundamental period of the structure and if the design response spectrum ordinates vary slowly over a wide range of periods that include the dominant modes of the structure. Both conditions are easily met for short-period hydraulic structures and smooth design response spectra with 5 percent damping or more. The CQC formula for the maximum combined displacements u_k to an earthquake in direction k is given by

$$u_k = \left[\sum_{i=1}^{N} \sum_{j=1}^{N} u_{ki} \rho_{ij} u_{kj} \right]^{1/2} \tag{2-6}$$

where u_{ki} and u_{kj} are the maximum modal displacements corresponding to the vibration modes i and j, respectively, and N is the number of modes. The cross-modal coefficients ρ_{ij} for the above two conditions and for the constant modal damping ξ are expressed by

$$\rho_{ij} = \frac{8\xi^2 (1+r) r^{3/2}}{(1-r^2)^2 + 4\xi^2 r (1+r)^2} \tag{2-7}$$

where r is the ratio of natural period of j^{th} mode, T_j, to the natural period of the i^{th} mode, T_i. As illustrated in Figure 2-9, this equation indicates a significant interaction between closely spaced modes, especially at high damping values.

(d) SRSS method. For the structures for which the modal periods are well separated, ρ_{ij} approaches zero (for $i \neq j$), and the CQC method degenerates into the familiar SRSS method. The maximum total response for a single earthquake response spectrum in direction k is then given by

$$u_k = \left[\sum_{i=1}^{N} u_{ki}^2 \right]^{1/2} \tag{2-8}$$

The SRSS method leads to conservative results for the well-separated vibration modes, but is inappropriate when they are closely spaced, because it ignores the contribution due to the cross-modal terms.

(e) Calculation of section forces and moments. The output of most computer response-spectrum analyses usually includes nodal displacements and element stresses only. The section forces and moments required for the design of structures are not readily available, except when beam elements are used to idealize the structure. For nonbeam elements, the combined maximum stress values have no sign, and thus evaluation of section forces from these stresses, if not interpreted properly, may lead to incorrect results. Like the element stresses, section forces and moments should first be evaluated for the individual modes and then combined by the CQC or SRSS method to obtain the maximum shears, thrusts, and moments at a specified cross section.

(f) Combining for multicomponent earthquake input. Three-dimensional structures are analyzed for three orthogonal components of the earthquake ground motions applied in two horizontal and vertical direction. The maximum response quantity of interest due to each component of the earthquake ground motion is estimated separately as described above. The maximum responses due to all three components of the earthquake ground motion are then combined according to the SRSS method:

$$u = \left[\sum_{k=1}^{3} u_k^2 \right]^{1/2} \tag{2-9}$$

The SRSS combination of the multicomponent earthquake responses can be used with either the SRSS or CQC modal combination method. With the SRSS modal combination, the summation for multi-component input can be performed before or after combination of the modal responses, but with the CQC method it should always be applied after the modal responses have been combined. Regular intake towers with a circular or rectangular cross section are usually analyzed for two horizontal components of the ground motion and not the vertical. For such towers in addition to the SRSS method, the peak value of any resultant response quantity due to the combined gravity and two horizontal components of ground motion may also be obtained from the largest of the values given by the following equations:

$$R = R_0 \pm R_x \pm \alpha R_y \tag{2-10}$$

$$R = R_0 \pm \alpha R_x \pm R_y \tag{2-11}$$

where R_x is the peak response due to the x-component; R_y the y-component of horizontal ground motion; and R_0, gravity loads. Equations 2-10 and 2-11 are usually used in conjunction with the standard response spectra, and the SRSS method is employed with the site-specific response spectra. The value of α for circular towers is taken equal to 0.40 and for rectangular towers equal to 0.3.

(g) Interpretation of analysis results. The basic results of response spectrum analysis consist of the maximum nodal displacements and element stresses (or forces). As discussed previously, these maximum responses are estimated by combining responses from individual modes and multicomponent input. The resulting dynamic responses obtained in this manner have no sign and may be interpreted as positive or negative. For example, the maximum element dynamic stresses σ_d are assumed to be either tension (positive) or compression (negative). Furthermore, the maximum values associated with each response quantity are not concurrent and usually occur at different instants of time. Thus static equilibrium checks cannot be performed to validate the results. Most computer programs used to perform response spectrum analysis do not compute section thrusts, shears, and moments for elements other than beam elements. To obtain section moments and forces from the computed stress results, the analyst should assign stress signs that would produce the correct stress distribution across a specified section. This is done by a careful examination of the shape of the predominant response modes, from which the actual deflected shape of a member and the associated stress distributions can be predicted. As discussed previously, a better approach is first to compute modal section forces and moments from the modal stresses, and then combine them using the CQC or SRSS method.

(h) Combining static and dynamic stresses. For the evaluation of earthquake performance of hydraulic structures, the response-spectrum estimate of the dynamic stresses σ_d should be combined with the effects of the static loads σ_s. Since response spectrum stresses have no sign, combination of static and dynamic stresses should consider dynamic stresses to be either positive or negative, leading to the maximum values of the total tensile or compressive stresses:

$$\sigma_{max} = \sigma_s \pm \sigma_d \tag{2-12}$$

It should be noted that only the similarly oriented components of σ_s and σ_d can be combined.

(3) Time-history method. The linear response of structures to earthquakes can also be computed using the time-history method of analysis. In the time-history analysis normally the acceleration time-histories are used as the seismic input. Procedures for developing acceleration time-history input consistent with the design response spectrum are described in Chapter 3. The idealized structural models used in the time-history analysis are essentially identical to those described previously for the response spectrum analysis. The response history is computed using a step-by-step numerical integration procedure applied either to the original equations of motion (direct method) or to the transformed equations in modal coordinates (mode superposition) (Bathe and Wilson 1976). In the more efficient mode superposition approach, first the response history for each mode is evaluated at each integration time-step, and then the modal response histories for all significant modes of vibration are combined to determine the dynamic response of the structure.

(4) Need for time-history analysis. Linear time-history analysis is required when the results of response spectrum analysis indicate that the computed maximum total stresses (or forces) exceed the allowable values, or when special conditions exist. The time-history analysis is performed to estimate the deformations and stresses (or forces) more accurately by considering the time-dependent nature of the dynamic response to earthquake motions. The results of such analysis serve to demonstrate the general behavior of the dynamic response, and combined with rational interpretation and judgment can provide a preliminary estimate of the level of inelastic behavior. Most Corps hydraulic structures are designed essentially to respond within their linear range when subjected to low to moderate intensity earthquakes. For this level of ground motions, the linear time-history analysis provides satisfactory results. For major earthquakes it is probable that the elastic capacity of the mass concrete would be exceeded, and some cracking and crushing of the concrete and yielding of reinforcing steels could occur. Prediction of the actual response and estimation of the expected damage and inelastic behavior under severe earthquakes can be evaluated only using a more complicated nonlinear analysis. However, linear analysis can still be very valuable for a preliminary assessment of the damage and the level of postelastic response and can help to decide whether a nonlinear analysis should be performed. As part of this evaluation, the results of linear analysis for the URC hydraulic structures should be examined in a systematic manner to identify the extent of overstressed regions at any particular point in time, to produce plots showing time-histories of stresses and other response quantities of interest, and to determine statistics on the number of stress cycles exceeding the allowable values and the corresponding excursions of these stress cycles beyond the specified limits.

b. Nonlinear time-history.

(1) Need for nonlinear analysis. A nonlinear time-history analysis may be necessary when the results of a linear analysis show that the structure could suffer significant damage during a major earthquake.

Minor local damages have little effect on the overall integrity of the structure and can still be evaluated by proper interpretation of the results of linear analysis. However, when the calculated tensile stresses (or forces) are significantly greater than the tensile strength of the concrete (or section capacity) over a large region and are repeated several times during the earthquake excitation, severe cracking of the concrete, joint slippage, and yielding of reinforcements can be expected. Under these conditions, the dynamic behavior of the structure is drastically different from the linear response, and a valid estimate of the damage is possible only if a true nonlinear performance is incorporated in the analysis.

(2) Realistic nonlinear analysis. A reasonable nonlinear analysis should take into account all sources of nonlinearity that contribute significantly to the nonlinear response behavior. The damage caused by earthquake shaking is normally associated with significant loss in the structural stiffness resulting from the concrete cracking, yielding of steel, opening of construction joints, slippage across the construction joints or cracking planes, and the nonlinear material behavior. Additional sources of nonlinearity arise from the nonlinear soil and the fractured foundation rock supporting the structure, as well as the separation of the structure and the foundation at the contact surface. At the present time, analytical techniques for a complete nonlinear earthquake analysis of hydraulic structures, including the interaction with foundation and water, are not available. Only limited aspects of the nonlinear behavior such as the contraction joint opening in arch dams, tensile cracking of gravity dams, sliding of blocks, and approximate postelastic analyses have been considered in practice. A realistic nonlinear analysis for the seismic safety evaluation of hydraulic structures depends on a great deal of new developments in the following topics: definition of seismic input, identification and specification of significant nonlinear mechanisms (joint opening and sliding, tensile cracking of the concrete, yielding and slippage of reinforcing steel, nonlinear material behavior under cyclic loads, etc.), development of idealized models representing the nonlinear behavior, numerical techniques and solution strategies for computing the nonlinear response, and development of criteria for acceptable performance and identification of possible modes of failure. The seismic input for a nonlinear analysis is in the form of acceleration time-histories. The key issues in developing time-histories for nonlinear analysis are duration of strong shaking, energy and pulse sequencing, special near-fault characteristics such as the source "fling," and the number of sets of time-histories required for the analysis. The main difficulty in effective nonlinear analysis at the present is the lack of or limited knowledge on the actual nonlinear material properties of the mass and reinforced concrete under cyclic loading.

2-9. Sliding and Rotational Stability During Earthquakes

a. Sliding stability. The sliding stability evaluation of hydraulic structures under earthquake loading can be made according to the traditional static equilibrium (seismic coefficient) and permanent displacement approaches described in Ebeling and Morrison (1992). In the traditional approach, the sliding stability is expressed in terms of a prescribed factor of safety against sliding, whereas in the permanent displacement approach the structure is permitted to slide along its base but the accumulated displacement during the ground shaking should be limited to a specified allowable value.

(1) Seismic coefficient approach. In the seismic coefficient approach, the safety against sliding is determined on the basis of shear-friction factor of safety (Ebeling and Morrison 1992). The shear-friction factor of safety is defined as the ratio of the resisting to driving forces along a potential failure surface

$$FS = \frac{CA + \left(\sum N - \sum U\right)\tan\phi}{\sum V} \tag{2-13}$$

where

C = unit cohesion

A = area of base

$\sum N$ = summation of normal forces

$\sum U$ = summation of uplift forces

tan ϕ = coefficient of internal friction

$\sum V$ = summation of shear or driving forces

(a) The driving forces acting on the structure include the static and seismic inertia forces due to weight of the structure and to hydrodynamic pressures. Treating the system above the failure surface as a rigid block, the inertia force associated with the mass of the structure is computed as the product of the seismic coefficient, as specified in ER 1110-2-1806, and the weight of the block. Similarly, the product of the seismic coefficient and the added mass of water moving with the structure produces inertia force due to the hydrodynamic pressure. The added mass of water may be computed using the Westergaard method (Westergaard 1933) or the equation given by Chopra (Chopra 1967):

$$M_a = 0.54\rho h^2 \qquad (2\text{-}14)$$

where ρ is the density and h is the depth of water. The motion of the structure relative to the failure surface is resisted by the shear strength mobilized between the structure and the surface by the friction and cohesion, as shown in Equation 2-13. For example, the shear friction factor of safety for sliding of the gravity dam shown in Figure 2-10 is given by

$$FS = \frac{CA + (W - U)\tan\phi}{\pm H_s + H_d + \left(\frac{W}{g}\right)a} \qquad (2\text{-}15)$$

where

W = weight of dam

H_s = hydrostatic force

H_d = hydrodynamic force (i.e., $M_a \cdot a$)

g = gravitational acceleration

a = ground motion acceleration or some fraction thereof

The ± sign is for sliding in the downstream or upstream direction with the plus sign indicating downstream.

(b) When the earthquake forces are included in the sliding stability analysis, the calculated factor of safety against sliding may become less than one. A factor of safety of less than one indicates a transient sliding. The sliding is assumed to occur for as long as the ground acceleration is greater than the critical value required for the driving force to exceed the resistance. However, due to the oscillatory nature of the earthquake ground motion, the sliding displacement is expected to be limited but could lead to excessive permanent displacements.

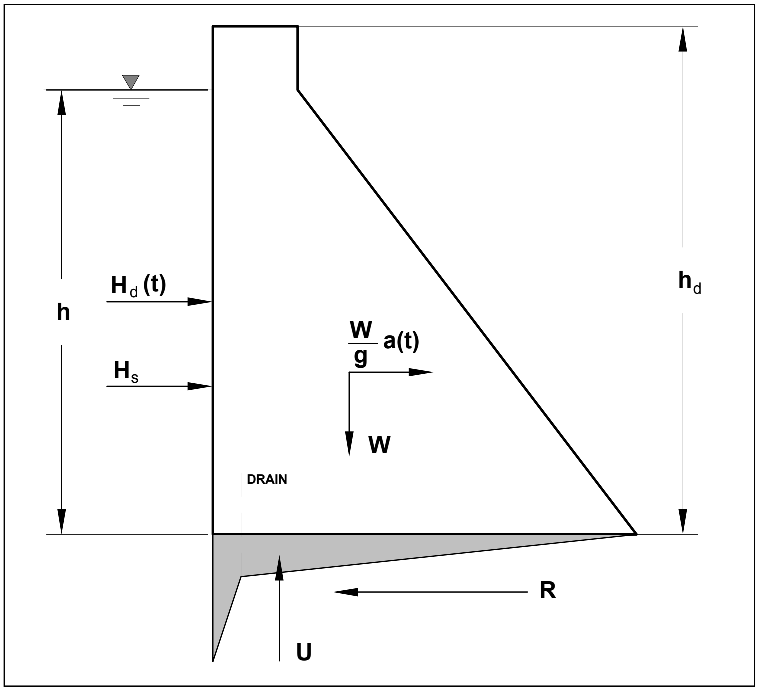


Figure 2-10. Forces acting on gravity dam monolith

(2) Permanent displacement approach. The traditional sliding stability criteria described above were developed for unrealistically small seismic forces based on a seismic coefficient of 0.1 or less. The factor of safety against sliding required by the traditional approach may not be attainable for larger seismic forces representative of the moderate- to high-intensity earthquake ground motions. In such cases, the sliding may occur but it takes place only during a short period of time associated with the acceleration cycles exceeding a critical acceleration, a_c, and diminishes during the remainder of these cycles when the acceleration is less than a_c and the relative velocity between the structure and the base is zero. Treating a gravity dam monolith as a rigid body supported on horizontal ground, and assuming that the motion of the dam relative to the ground is resisted by the friction between the dam and the ground surface, the critical acceleration a_c is given by (Chopra and Zhang 1991)

$$\frac{a_c}{g} = \frac{1}{W + W_{a0}}\left[\mu_s\left(W - U\right) \pm H_s\right] \tag{2-16}$$

where μ_s is the coefficient of static friction, and W_{a0} is the weight of water which represents the hydrodynamic force. The ± sign in this equation is for sliding in the upstream or downstream direction. It is apparent from Equation 2-16 that the critical acceleration required to slide the dam downstream is smaller than that needed to move the dam upstream. Similarly, the critical acceleration a_c necessary to initiate sliding in other hydraulic structures can be derived by considering the equilibrium of forces involved in each particular structure. Knowing the critical acceleration a_c, the permanent sliding displacements can be estimated using the Newmark's rigid block model (Newmark 1965). According to Newmark's concepts, also discussed by Chopra and Zhang (1991), the upper bounds for permanent displacements of the sliding rigid mass subjected to earthquake ground motion with peak velocity v_m and peak acceleration a_m can be estimated from :

$$s_m = \frac{v_m^2}{2a_c}\left(1-\frac{a_c}{a_m}\right)\frac{a_m}{a_c} \qquad (2\text{-}17)$$

$$s_m = \frac{v_m^2}{2a_c}\cdot\frac{a_m}{a_c} \qquad (2\text{-}18)$$

$$s_m = \frac{v_m^2}{2a_c}6 \qquad (2\text{-}19)$$

These equations, plotted in nondimensional form in Figure 2-11, show that Equation 2-18 provides more conservative values than Equation 2-17, and Equation 2-19 is intended for systems with the small values of a_c/a_m. The portion of the curve for each equation where the equation is recommended for use is a thick line. Newmark's model provides an easy means for approximate estimation of the upper bounds for permanent sliding displacements, but it is based on certain assumptions that ignore the true dynamic response behavior of the sliding. More accurate estimates of the sliding displacements can be made from the response history analysis proposed by Chopra and Zhang (1991).

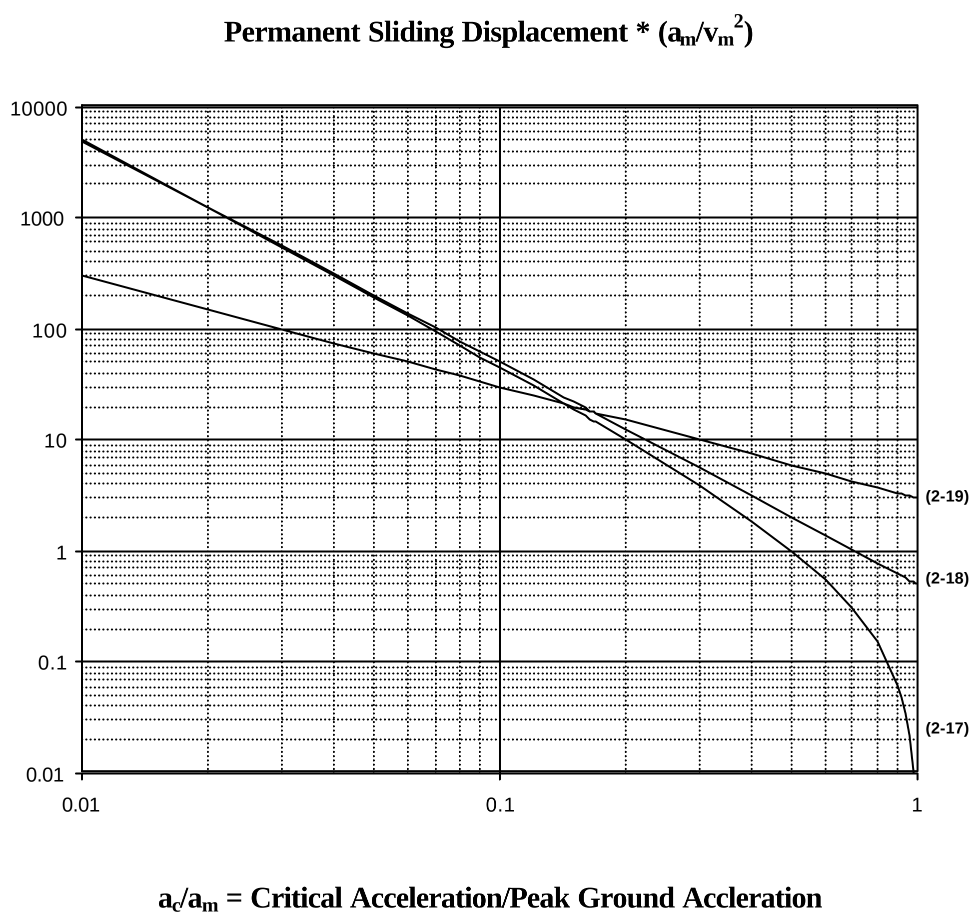


Figure 2-11. Newmark's upper bounds for permanent sliding displacement (Chopra and Zhang (1991), courtesy of Earthquake Engineering Research Center, University of California, Berkeley)

b. Rotational stability. Hydraulic structures subjected to large lateral forces produced by major earthquakes may tip and start rocking when the resulting overturning moment becomes so large that the structure breaks contact with the ground.

(1) Intake towers. For an intake tower idealized as a nearly rigid or flexible equivalent SDOF system (Figure 2-12), the tipping occurs when the overturning moment exceeds the resisting moment due to the weight of the structure. This condition is expressed by:

$$mS_a h > mgb \tag{2-20}$$

$$S_a > g\,(b/h) \tag{2-21}$$

where

m = mass of structure

S_a = spectral acceleration of the earthquake ground motion

h = one-half height of structure

b = one-half base width of structure

Similar expressions can also be derived for other hydraulic structures, except that the moments due to hydrostatic and hydrodynamic forces should be included ((2) below). In both cases it is assumed that the structure is not bonded to the ground, but it may be keyed into the soil with no pulling resistance. It should be noted that the structure will eventually overturn if the moment $M > mgb$ is applied and sustained, where *mgb* represents the resisting moment due to the weight of the structure. However, under earthquake excitation large overturning moments occur for only a fraction of a second in each cycle, with intermediate opportunities to unload. By comparing the earthquake average energy input with the required average energy for overturning the structure, Housner provided the following relationship as a criterion for the rotational stability of a rocking structure (Housner 1963):

$$\alpha = S_v \sqrt{\frac{mr}{gI_0}} \tag{2-22}$$

where

α = an angle defined in Figure 2-12

S_v = spectral velocity of the earthquake ground motion

m = mass of structure

r = radial distance from the center of gravity to tipping edge

I_0 = moment inertia about the corner

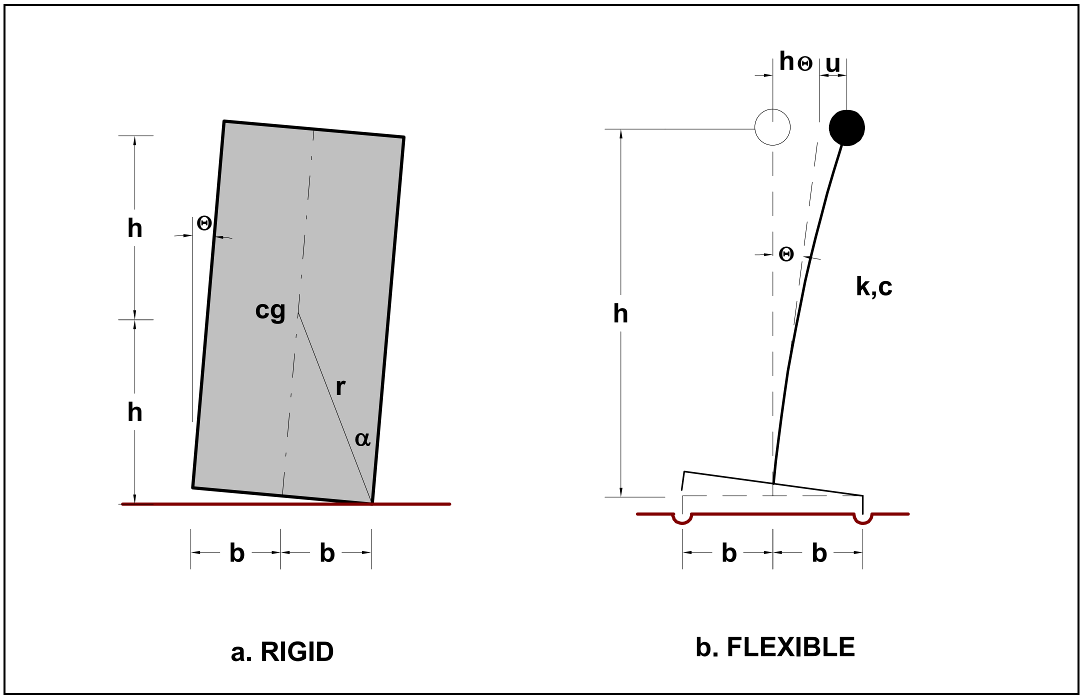


Figure 2-12. Rigid block and SDOF models for rigid and flexible structures

Based on the average energy formulation used, this equation is interpreted as stating that for a given spectral velocity S_v, a block having an angle of α given by Equation 2-22 will have approximately a 50 percent probability of being overturned (Housner 1963). For slender structures such as intake towers, Equation 2-22 can be approximated by

$$\alpha = \frac{S_v}{\sqrt{gr}} \tag{2-23}$$

By combining Equations 2-21 and 2-23 and using the relationships among the spectral acceleration, velocity, and displacement, Scholl (Applied Technology Council (ATC) 1984) found that consideration of one spectral parameter alone as the earthquake demand is not sufficient for evaluating overturning and suggested the following relationships:

$$S_d = b \quad \text{when} \quad S_a = g\frac{b}{h} \tag{2-24}$$

These equations show that when S_a is just sufficient to cause tipping, the structure will start rocking, but its displacement approximated by spectral displacement S_d must reach the value of b before it can overturn. These equations also demonstrate why larger structures such as buildings do not overturn during earthquakes, whereas smaller rigid blocks having the same aspect ratios are expected to overturn. This is because, in general, S_d is never large enough to tip over a building, but it can approach the one-half base width (i.e., b) of smaller rigid blocks such as tombstones.

(2) Gravity dams. A preliminary study of the rotational stability of a gravity dam may be carried out as described by Chopra and Zhang (1991). The dam is assumed to be rigid and subjected to both the horizontal and vertical components of earthquake ground motion. The dam starts tipping in the

downstream direction if the overturning moment due to upstream ground acceleration and water pressures exceeds the restoring moment (Figure 2-13) as follows:

$$M\left|a_x(t)\right|y_c + M_d(t) + \frac{1}{3}hH_s + U\left(b - b_U\right) \geq M\left[g + a_y(t)\right]\left(b - x_c\right) \tag{2-25}$$

where parameters are shown in Figure 2-13 and $M_d(t)$ is the moment due to hydrodynamic pressure given by

$$M_d(t) = -M_{d0}a_x(t) + \frac{1}{6}\rho h^3 a_y(t) \tag{2-26}$$

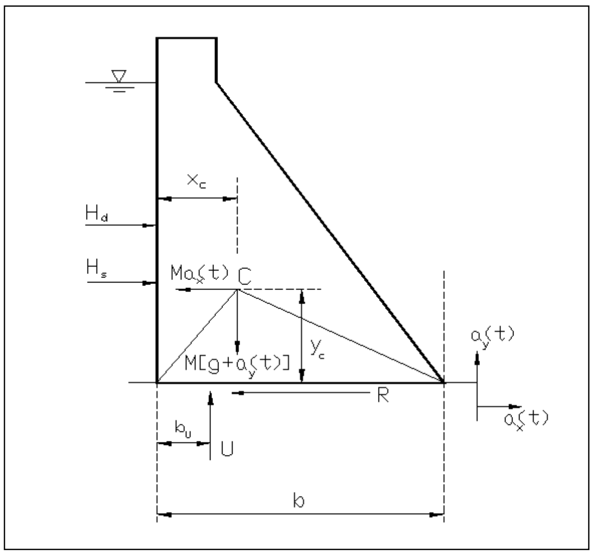


Figure 2-13. Rigid gravity dam on horizontal ground

The first term, $M_{d0} = \int_0^h p_0(y)y\,dy$, is moment due to hydrodynamic pressure generated by the horizontal ground motion. The pressure $p_0(y)$ may be obtained using either the Westergaard (1933) or Chopra (1967) formula. The second term in Equation 2-26 is the moment due to hydrodynamic pressure, $\rho(h - y)$ a_y (t), produced by the vertical motion. Substituting Equation 2-26 into 2-25 gives the critical upstream acceleration a_c required to initiate downstream tipping of the dam about its toe:

$$a_c = \frac{1}{My_c + M_{d0}} \times \left\{\left[M\left(b - x_c\right) - \frac{1}{6}\rho h^3\right]\left[g + a_y(t)\right] - U\left(b - b_u\right)\right\} \tag{2-27}$$

Similarly, tipping about the heel of the dam initiates when the critical downstream acceleration a_c reaches

$$a_c = \frac{1}{My_c + M_{d0}}\left\{\left(Mx_c + \frac{1}{6}\rho h^3\right)\left[g + a_y(t)\right] - Ub_u\right\} \tag{2-28}$$

When subjected to strong ground motions, a rigid dam may move with ground, slide only, rock only, and rock and slide. Comparison of critical accelerations for sliding and tipping can show which motion will start first. The parameter studies by Chopra and Zhang (1991) over a wide range of parameters indicate that downstream sliding of the dam will initiate before tipping or upstream sliding. Large downstream accelerations will usually cause upstream tipping of the dam about its heel before upstream sliding. In most cases, the more likely type of motion is the downstream sliding of the dam. The effect of rocking on the sliding motion is considered to be negligible and may be ignored in the evaluation of sliding response.

2-10. Current Practice on Use of Response Spectra for Building-Type Structures

a. General. The design requirements of the Structural Engineers Association of California (SEAOC) are the most current state of practice for the earthquake-resistant design of buildings in California. SEAOC also includes provisions for lower seismic hazard regions in California, which may be suitable for use in regions outside of California. A summary of the SEAOC's use of design response spectra and the similarities and differences between the SEAOC's recommendations and the procedures used for the hydraulic structures are described in this section.

b. Criteria for dynamic analysis. According to SEAOC's recommendations, dynamic analysis procedures should be used for the design and analysis of certain building structures. This includes buildings 73.2 m (240 ft or more) in height, except for those located in Seismic Zone 1 and standard occupancy structures in Zone 2, buildings with irregular stiffness, mass, or vertical geometry, buildings over five stories or 20 m (65 ft) in height located in Seismic Zones 3 and 4, or buildings founded on soft soils (type S4) with fundamental period of vibration greater than 0.7 sec. The dynamic analysis procedures for buildings are based on the same general concepts described for the hydraulic structures. The response spectrum analysis is the preferred method for most buildings. The time-history analysis is employed to study inelastic response characteristics or to incorporate time-dependent effects in the elastic dynamic response. However, structural modeling, design earthquakes, and acceptable level of nonlinear response for buildings are different from those for hydraulic structures.

c. Structural modeling. The obvious differences between building and hydraulic structures are the structural system and the function. While buildings are made primarily of frame systems with shear walls and braces, hydraulic structures are built as massive plain or lightly reinforced concrete monoliths to contain or retain water. The idealized model of a building usually consists of beam and column elements with the mass of the building lumped at a few selected nodes. Most regular buildings can be adequately idealized by one- or two-dimensional models. Very complex and highly irregular buildings or those with large eccentricities between the center of mass and resistance require 3-D analysis. The majority of hydraulic structures, on the other hand, are modeled as planar or 3-D models using 2-D or 3-D solid and shell elements as discussed previously. Furthermore, interactions with the water and foundation are important aspects of the dynamic characteristics of hydraulic structures that need to be included in the analysis.

d. Design response spectra. Hydraulic structures and civil works buildings are analyzed for two levels of design earthquakes (as described previously and in reference to ER 1110-2-1806, respectively). SEAOC requires a single design earthquake that as a minimum should have a 10 percent probability of exceedance in 50 years, which gives a return period of 475 years. The ground motion for buildings may be represented by normalized response spectra, site-specific response spectra, or time-histories. The normalized response spectra, shown in Figure 2-14, are permitted for the soil profiles S_1, S_2, and S_3 defined in Table 2-1. Site-specific response spectra are required if the site condition significantly differs from those used to develop the normalized shapes, or if the structure is founded on soil profile S4, or the structure is seismically isolated. The time-histories of the ground motions are usually developed for inelastic analysis and for the seismic-isolated structures. The probabilistic seismic hazard analysis is commonly used to develop site-specific response spectra for building sites in Zones 3 and 4.

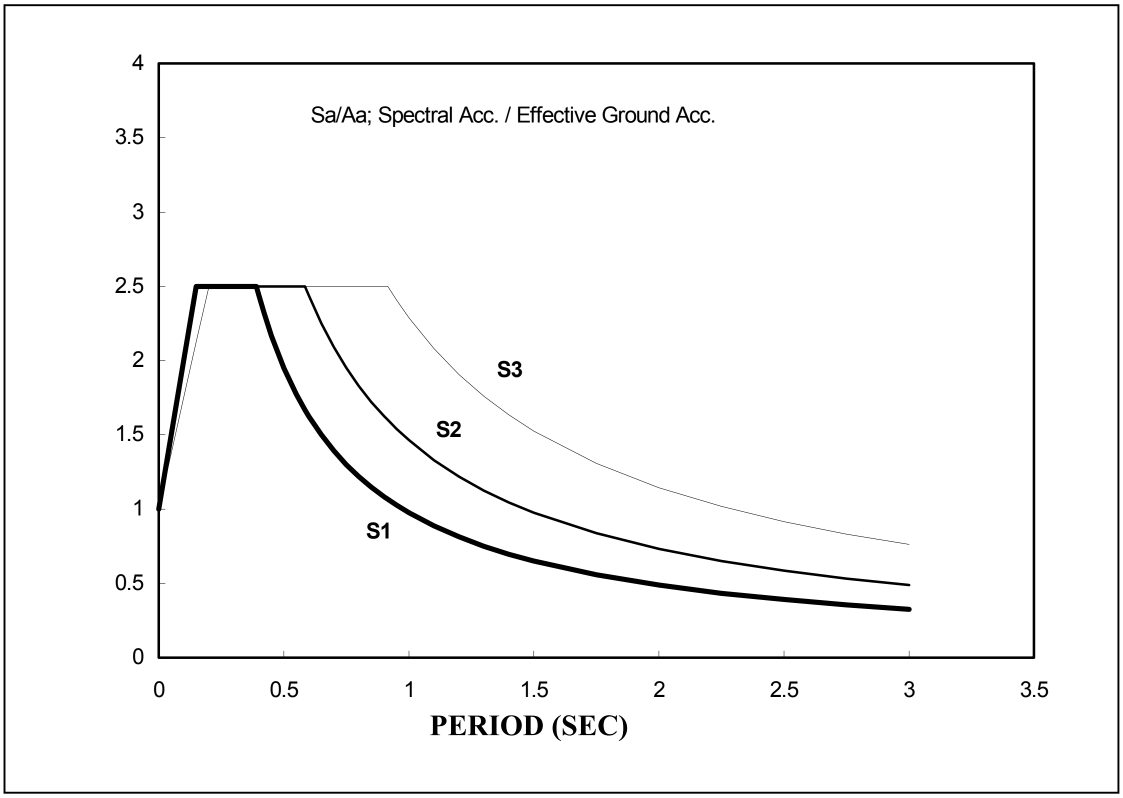


Figure 2-14. Normalized SEAOC's design response spectra

Table 2-1
SEAOC's Site Coefficients

Type	Description	S Factor
S1	A soil profile with either: 1 A rock-like material characterized by a shear wave velocity greater than 762 m/sec (2,500 fps) or by other suitable means of classification, or 2 Medium-stiff to stiff or medium dense to dense soil conditions where soil depth is less than 70 m (200 ft).	1.0
S2	A soil profile with predominantly medium dense to dense or medium stiff to stiff soil conditions where soil depth exceeds 70 m (200 ft) or more.	1.2
S3	A soil profile containing more than 6 m (20 ft) of soft to medium stiff clay but not more than 12 m (40 ft) of soft clay.	1.5
S4	A soil profile characterized by a shear wave velocity less than 152 m/sec (500 fps) and containing more than 12 m (40 ft) of soft clay.	2.0

1 Reprinted from SEAOC's Recommended Lateral Force Requirements and Commentary, 1996 (SEAOC 1996).

e. Nonlinear response of buildings. The design practice for buildings permits the structure to respond inelastically during a major earthquake but suffer only acceptable and predictable amounts of damage without collapse. The energy dissipated through such inelastic deformations is utilized to reduce the level of seismic design forces. According to the SEAOC's Commentary, structures designed in conformance with its recommendations should, in general, be able to

(1) Resist a minor level of earthquake ground motion without damage.

(2) Resist a moderate level of earthquake ground motion without structural damage, but possibly experience some nonstructural damage.

(3) Resist a major level of earthquake ground motion having an intensity equal to the strongest either experienced or forecast at the building site, without collapse, but possibly with some structural as well as nonstructural damage. To achieve such performance goals, SEAOC recommends that the design forces be determined by reducing the elastic forces obtained from linear analysis by the system quality factor R_w. The structural system quality factor R_w represents the overall ductility and energy dissipation capacity of the system when strained beyond its elastic limit. The R_w values are selected according to the ability of a particular system to sustain cyclic inelastic deformations without collapse. Factors contributing to the actual selection of R_w include redundancy, reliability of as-built performance, inelastic load-deformation behavior, and changed damping and period modification with deformation. Experience indicates that buildings designed based on this procedure have exhibited adequate performance in most cases. In addition to the ductility requirement, most seismic codes require a limitation on the story drift to limit nonstructural damage during more frequent earthquakes and to ensure building stability under the major earthquakes. For tall buildings, the drift limitations may dictate an elastic design, even for MCE ground motions.

f. Inelastic design response spectra. For regular buildings, the code specified design seismic forces can be estimated from

$$V = C_s W = \frac{(S_a/g)}{R_w} \cdot W \tag{2-29}$$

where

V = base shear

C_s = design seismic coefficient

W = weight of building

S_a = linear elastic response spectral acceleration

R_w = the structural system quality factor intended to account for ductility and energy dissipation when the structure deforms beyond the yield point

The design seismic coefficient C_s can be obtained from a reduced response spectrum that can be considered to be the inelastic design response spectrum (IDRS). However, recent statistical studies have shown that the shape of IDRS significantly differs from the shape of elastic response spectra (Miranda 1992; Krawinkler and Rahnama 1992), and is strongly influenced by the level of inelastic deformation, local site conditions, and the period of vibration. Until more data become available, the most reliable procedure for establishing an IDRS is to perform nonlinear dynamic time-history analyses of structures with different degrees of ductility ratios using the available recorded ground motions.

g. Nonlinear inelastic response of reinforced concrete hydraulic structures.

(1) Free-standing intake towers. The response of a free-standing intake tower to earthquake ground shaking is similar to that of a cantilever wall that resists earthquake forces by flexural and shear deformations. Thus, the principles of inelastic design of reinforced concrete walls are generally applicable to free-standing towers, provided that the response is consistent with the post-yield capacity of reinforced tower structure. Similar to the structural walls with limited ductility, intake towers should be designed such that flexural yielding controls the strength, the inelastic deformations, and thus the energy dissipation in the entire structure (Pauly 1986). Failure modes due to diagonal tension or diagonal compression caused by shear and sliding shear along the base of the structure should be avoided. The main source of energy dissipation should be yielding of flexural reinforcement in the region where the plastic hinges are expected to develop. Recent research at CEWES has demonstrated that a lightly reinforced rectangular intake tower possesses sufficient ductility to allow formation of a plastic hinge at the base of the tower with a limited amount of inelastic behavior. The ability of such plastic hinges to sustain the repeated cycles of inelastic demands imposed on them, the expected length of the plastic hinge, and the acceptable level of damage during major earthquakes are being investigated at CEWES. Until these have been established, preliminary design and screening evaluations for retangular intake towers should be limited to an inelastic response not greater than twice the yield deflection in accordance with the design provisions of EM 1110-2-2400.

(2) Other reinforced concrete hydraulic structures. For other reinforced concrete hydraulic structures with structural configurations and systems different from those of buildings and intake towers, postelastic analyses should be performed to establish appropriate ductility for the members so that the structure will undergo controlled levels of nonlinearity without compromising structural safety. Designing for to MDE take advantage of the ability of the structures to dissipate energy through inelastic deformations should produce uniform patterns of yielding and energy absorption. While the initial forces are distributed through the structure according to the elastic stiffness of the various members and connections, partially developed plastic hinges in the critical members will redistribute forces to the stiffer members and may not follow the load paths envisioned in the initial design. If the desired level of ductility is not reached or the forces are not distributed appropriately, then it is possible that the actual member forces may exceed the design values. In addition, the possibility that all cycles of nonlinear response (i.e., hysteretic energy demand), and not just the maximum response cycle, may cause damage should also be investigated. In other words, it is important to note that the specified ductility may be reached either once or several times during the ground shaking, and to ensure that the lightly reinforced concrete hydraulic structure has sufficient ductility to resist such repeated demands. These considerations indicate that the inelastic design of hydraulic structures requires careful attention to the actual postelastic behavior of the structure and should be done in consultation with and approved by CECW-ET.